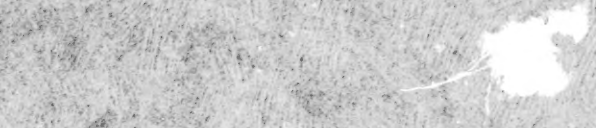
SHR-83-06
Revised

Design of Wood Bridges Using the Ontario Highway Bridge Design Code



SRR-83-02
Revised

Design of Wood Bridges Using the Ontario Highway Bridge Design Code

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Title

DESIGN OF WOOD BRIDGES USING THE ONTARIO HIGHWAY BRIDGE DESIGN CODE REVISED MARCH 1984

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Abstract

The Ontario Highway Bridge Design Code (OHBDC) [1] is the first bridge code in North America which is based entirely on the concept of Limit States Design (LSD). The second edition of the OHBDC [2] unlike the first edition, which was introduced on a trial basis, will become the only accepted code for the design of highway bridges in Ontario.

The objective of this report is to familiarize the designer with the new wood design specifications of the OHBDC. Using a design example with direct references to the specifications enables the discussion of a number of important design considerations.

The recent trend towards rehabilitation has brought many existing wood bridges into the foreground. Therefore, this report also includes some discussion regarding the inspection and evaluation of existing bridges with respect to the OHBDC.

Key Words Design, Bridges, Wood, OHBDC, Example

Comments This is a revised printing which is updated to comply with the "as-published" OHBDC specifications.

Distribution Unrestricted, MTC HO + regions, selected structural and geotechnical engineers.

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TABLE OF CONTENTS

PAGE

1. INTRODUCTION 1

2. OHBDC SPECIFICATIONS 2

3. DESIGN EXAMPLE 4

4. EVALUATION OF EXISTING BRIDGES 19

5. CONCLUSIONS 23

6. REFERENCES 24

LIST OF FIGURES PAGE

Figure 1 Design Example 3

Figure 2 Loading for Laminated Deck 5

1. INTRODUCTION

The Ontario Highway Bridge Design Code (OHBDC) [1] is the first bridge code in North America which is based entirely on the concept of Limit States Design (LSD). The second edition of the OHBDC [2] unlike the first edition, which was introduced on a trial basis, will become the only accepted code for the design of highway bridges in Ontario.

The introduction of LSD into wood design was still in it's early stages in North America at the time of the development of the first edition of the OHBDC. The data-base available for the development of the specifications on wood design in the first edition was very limited. As a result the first edition was not to be applied to the design of wood bridges.

Considerable development work has since been performed which has enabled the formulation of a new set of wood design specifications. These specifications, which have been introduced in the second edition of the OHBDC, are based primarily on the LSD concept.

The objective of this report is to familiarize the designer with the new wood design specifications of the OHBDC. Using a design example with direct references to the specifications enables the discussion of a number of important design considerations.

The recent trend towards rehabilitation has brought many existing wood bridges into the foreground. Therefore, this report also includes some discussion regarding the inspection and evaluation of existing bridges with respect to the OHBDC.

2.0 OHBDC SPECIFICATIONS

It is assumed that the designer is familiar with the LSD concept and Simplified Methods of Analysis as applied by the OHBDC. Only in some cases where the specifications differ from those of the first edition or apply only to wood design does this report provide additional discussion on other sections of the OHBDC.

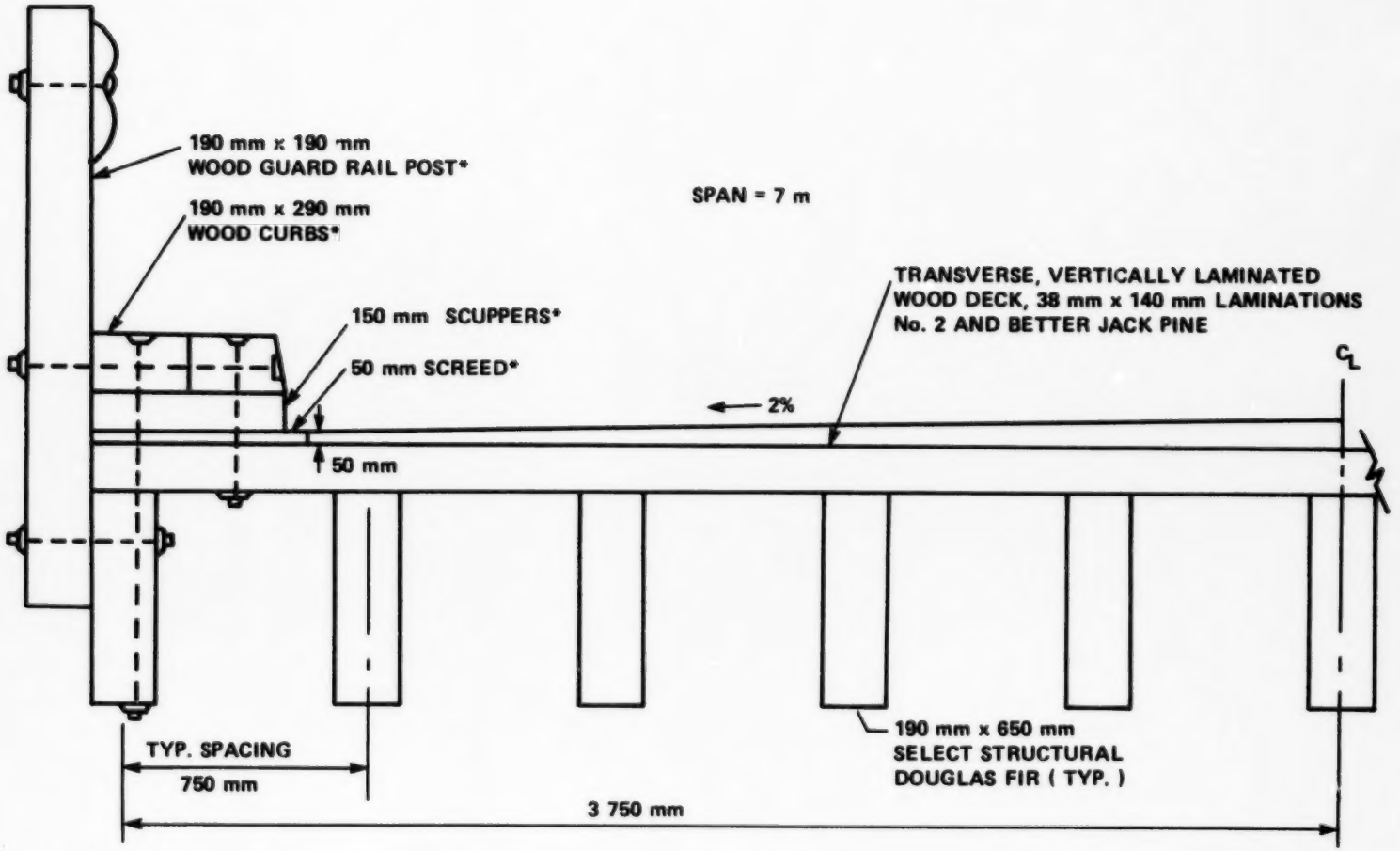
2.1 Definitions and Notations

During the development of the wood specifications for the OHBDC it became apparent that a number of terminologies in the area of wood design offered varied interpretations. In order to establish defined terminologies Section 13.2 of the wood section provides a list of definitions. It is recommended the designer become familiar with these definitions in order to ensure proper interpretation of the design specifications.

In addition, all variables provided in the test are defined in Section 13.3 on Notation. Included in the notation are the units which are assumed in the applications within the specifications. Though in many instances the designer may choose to use other units, as long as they are consistent, it should be done with caution as several formulae are unit dependent.

2.2. General Considerations

In the process of developing the OHBDC wood design specifications considerable new research and development has been implemented. Though the OHBDC specifications are similar to previous bridge design codes, in that they provide for nearly all of the same design considerations, the basis for some design criteria has changed. Rather than review all the areas of change the designer is referred to the commentary to Section 13. It provides a Clause by Clause review of the specifications and includes a very extensive list of references.



* CURB AND GUARD RAIL DETAILS REPRESENT STANDARD PRACTICE BUT SHOULD NOT BE TAKEN AS MEETING THE OHBDC REQUIREMENTS

Figure 1/ Design Example

3.0 DESIGN EXAMPLE

The following design example involves the design check of the structure depicted in Figure 1. This includes evaluation of the laminated decking and sawn wood stringers.

3.1 Transverse Laminated Wood Decking

The decking material is specified as 38 mm x 140 mm Jack Pine, graded as No.2 and better. This grading is in keeping with the ready availability of the dimension lumber sizes which are normally supplied as a mixed grade. This grading assumes a maximum of 50 percent No. 2 and the rest No. 1 and Select Structural. This is a recommended material specification for laminated decking, though the designer may choose to use only one grade. However usually there is little benefit in separating the grades and the supplier will supply the requested grade and/or better.

3.1.1 Specifications

Section 13.7 of the OHBDC indicates that shear need not be considered for laminated decks. Therefore at ULS only the flexural capacity of the deck is considered.

The governing load case for the laminated deck is ULS Combination 1 (Table 2.5.1(a)). However, only the live load and dynamic load allowance will be considered as the dead load is not significant. The critical loading and distributions are shown in Figure 2 using the maximum 100 kN wheel of the OHBDC design vehicle. The load is assumed to be distributed over the width of the wheel in the direction of the span. The width of decking, perpendicular to the span, which is assumed to carry the load is 400 mm as specified in Section 3.7.1.3.3. This section also defines the span as the clear distance, plus one half of one flange of the main girder and also allows the designer to assume the maximum moment is 0.8 of the maximum simple span moment.

The dynamic load allowance (DLA) for components governed by a single axle is 0.40 (2.4.3.2.3), however for wood components this is modified by 0.7 (2.4.3.2.10). Therefore the final DLA is 0.28.

NOTE: All underlined references are with respect to the OHBDC.

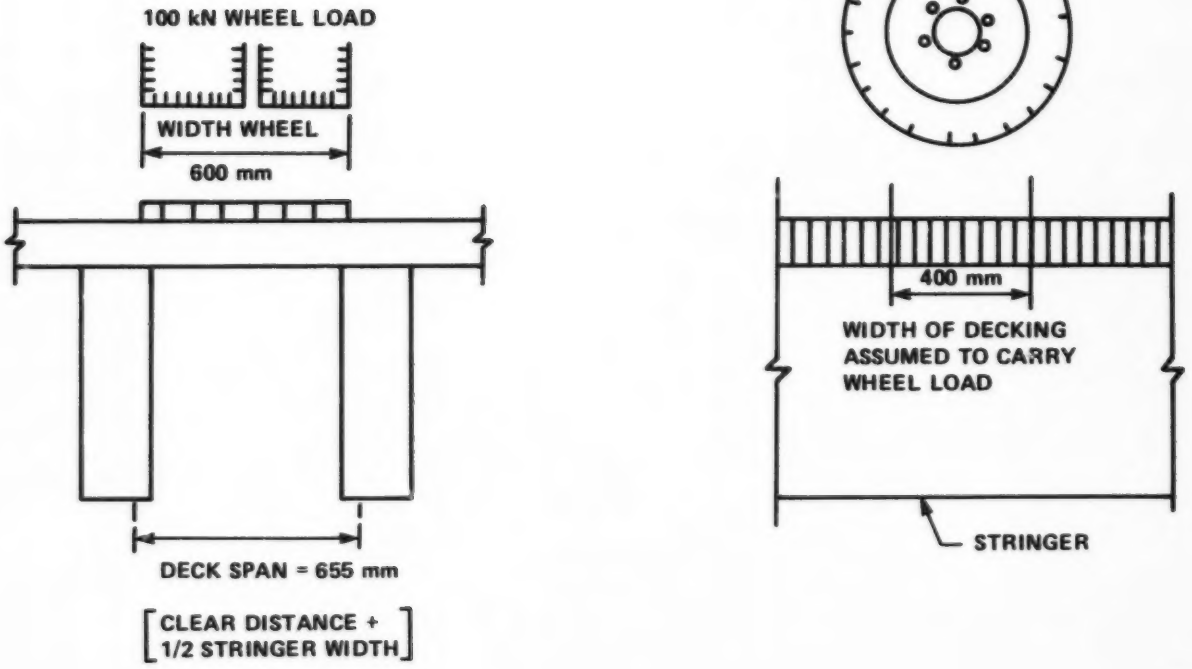


Figure 2/ Loading for Laminated Deck

All other design criteria are referenced in the following design calculations.

3.1.2. Design

Span ℓ = 655 mm

Dynamic Load Allowance DLA = 0.28

Load Factor α = 1.4 (2.5.1(a))

Total Factored Load Effect "M"

total factored wheel load

= α (100 000 N) (1 + DLA)

= 1.4 (100 000 N) (1 + 0.28) = 179 200 N

This can be represented as a distributed load over the width of the wheel,

$$w = \frac{179\ 200\ N}{600\ mm} = 299\ N/mm$$

maximum moment = 0.8 (maximum simple span moment)

therefore total factored load effect is

$$M = 0.8\ \frac{(w\ell^2)}{8} = 0.8\ \frac{(299\ N/mm)\ (655\ mm)^2}{8}$$

$$M = \underline{\underline{12.8 \times 10^6}}\ N - mm$$

(this is conservative as the length of the UDL is 600 mm not 655 mm)

Factored Resistance M_u

$M_u = \phi\ f_{bu}\ S\ k_d\ k_\ell\ k_m\ k_s$ (13.6.1.)

where: ϕ = performance factor = 0.9 (13.4.4.)

f_{bu} = specified strength = 11.5MPa (Table 13.11.2.)

S = section modulus = 3.25×10^6 mm^3 /m width

k_ℓ = slenderness effect NOT APPLICABLE

k_m = load sharing = 1.2 (13.5.6.)

k_s = size effect = 1.3 (13.6.2.1.)

therefore $M_u = 0.9\ (11.5MPa)\ (3.25 \times 10^6\ mm^3)\ (1.2)\ 1.3$

$M_u = 52.5 \times 10^6$ N - mm per m width deck

therefore over the 400 mm distribution width the total Factored Resistance

$M_u = 0.4\ (52.5 \times 10^6\ N - mm)$

$M_u = \underline{21\ kN - m} > 12.8\ kN - m$

therefore decking is okay.

3.2 Longitudinal Sawn Wood Stringers

Few species of structural softwood are capable of supplying the large sizes required for sawn wood stringer bridges. In Ontario the most common is Douglas Fir in either Select Structural or No. 1 grade.

Using Select Structural Douglas Fir the material properties for design are;

Specified Strengths		(Table 13.11.2(b))
flexure		$f_{bu} = 24.0$ MPa
shear		$f_{vu} = 1.1$ MPa
modulus of elasticity		$E_{50} = 11\ 000$ MPa
shear modulus		$G = 0.065E = 715$ MPa
modulus of elasticity decking	$E_{50} = 9500$ MPa	(Table 13.11.2(a))

The modulus of elasticity and shear modulus are required for the simplified analysis, which also requires the modulus of elasticity of the decking.

3.2.1 Ultimate Limit States

This design example will consider only the ULS Combination 1 which will include live load, DLA and dead load. However, the designer must evaluate all the ULS combinations to determine the critical case.

3.2.1.1 Flexure

Factored Flexural Resistance M_u

$$M_u = \phi f_{bu} S k_d k_\ell k_m k_s$$

where $\phi = 0.9$ (Table 13.4.4.)

$f_{bu} = 24.0$ MPa

$S = \frac{bd^2}{6} = \frac{190(650)^2}{6} = 13.38 \times 10^6$ mm^3

$k_d = 1.0$ (13.5.3.)

$k_m = 1.0$ (13.5.6.)

$k_s = 0.8$ (13.6.2.1.)

$k_\ell = 1.0$ (13.6.4.3.)
a laminated wood deck is considered to provide lateral support to the compression zone of the stringer.

therefore the factored flexural resistance is

$$M_u = 0.9\ (24.0\ \text{MPa})\ (13.38 \times 10^6\ \text{mm}^3)(0.8)$$

$$M_u = \underline{\underline{231 \times 10^6\ \text{N - mm}}} \text{ / stringer}$$

Load Distribution - Simplified Methods

The following additional information is required to apply the simplified method of analysis outlined in Section 3.7 of the OHBDC.

Design lane width $w_e = w_c/n$ (2.4.3.1.4.)

where w_c = curb to curb width = 6.53 m (Figure 1)

$n = 2$ (Table 2.4.3.1.4.)

therefore $w_e = \frac{6.53}{2}\ \text{m} = 3.27\ \text{m}$

Idealization of Structural Parameters (Table A3.2(d))

longitudinal flexural rigidity D_x

$$D_x = E_L \frac{b\, t_2^3}{12\, P_x}$$

where $E_L = E_{50}$ (stringer) = 11000 MPa

b = width stringer = 190 mm

t_2 = depth stringer = 650 mm

P_x = stringer spacing = 750 mm

therefore $D_x = 1100 \frac{(190)\,(650)^3}{12(750)}$

$$D_x = 6.38 \times 10^{10} \text{ N - mm}$$

Transverse flexural rigidity D_y

$$D_y = E_L \frac{t_1^3}{12}$$

where $E_L = E_{50}$ (decking) = 9 500 MPa

t_1 = deck depth = 140 mm

therefore $D_y = 9500 \frac{(140)^3}{12}$

$$D_y = 0.217 \times 10^{10} \text{ N - mm}$$

Longitudinal torsional rigidity D_{xy}

$$D_{xy} = G_{LT} \frac{k\, t_2\, b^3}{P_x}$$

where G_{LT} = shear modulus (stringer) = 715 MPa

k = from Table A3.2(c) for a/b

where a = widest dimension = 650 mm

b = narrowest dimension = 190 mm

therefore $a/b = 3.42$ and $k = 0.271$

therefore $D_{xy} = 715 \frac{(0.271)\,(650)\,(190)^3}{750}$

$$D_{xy} = 0.115 \times 10^{10} \text{ N - mm}$$

Transverse torsional rigidity $D_{yx} = 0.0$

and Coupling rigidities $D_1 = D_2 = 0.0$

Simplified Method of Analysis

longitudinal bending moments (3.7.1.2.1.)

$$\alpha = \frac{D_{xy} + D_{yx} + D_1 + D_2}{2\,(D_x\, D_y)^{0.5}}$$

$$= \frac{0.115}{2(638 \times 0.217)^{0.5}}$$

$$\alpha = 0.0489$$

$$\Theta = \frac{b}{L}\left(\frac{D_x}{D_y}\right)^{0\ 25}$$

where $b = \frac{1}{2}$ width bridge $= 3.85$ m

$L =$ span $= 7.0$ m

therefore $\Theta = \frac{3.85}{7.0}\left(\frac{6.38}{0.217}\right)^{0.25}$

$$\Theta = 1.28$$

$$\mu = \frac{w_e - 3.3}{0.6} \neq 1.0$$

$$= \frac{3.27 - 3.3}{0.6} = -.05$$

Select distribution factors from Table 3.7.1.2(a)

"Governing Load Case in Two Lane Bridges"

$D = 1.86$ for external portions

$D = 1.57$ m for internal portions

$C_f = 5.0$

therefore final D_d values

$$D_d = D\left(1 + \frac{\mu C_f}{100}\right)$$

$$D_d = 1.86 \left(1 + \frac{(-.05)\ (5)}{100}\right) = 1.86 \quad \text{external portions}$$

$$D_c = 1.57 \left(1 + \frac{(-.05)\ (5)}{100}\right) = 1.57 \quad \text{internal portions}$$

therefore internal $D_d = 1.57$ is critical

Note: this distribution factor already includes modification for multi-lane loading (3.7.1.2.1(f)).

The governing load fraction of one line of wheels of the OHBDC design vehicle (or half the design lane loading) is:

$$\frac{S}{D_d} = \frac{0.75 \text{ m}}{1.57 \text{ m}} = \underline{0.48}$$

Live Load "M_{LL}"

Maximum bending moment per design lane for 7 m span is 410 kN - m due to the OHBDC design vehicle (Fig. 2.4.3.1.2(a)) therefore unfactored live load moment per ½ design lane is 205 kN - m

$$M_{LL} = 205 \text{ kN - m}$$

Dead Load "M_{DL1}" and "M_{DL2}"

Assume 100 mm asphalt for internal portions where unit weight is 23.5 kN/m^3 (2.4.2.2.)

therefore $w_A = .1 \text{ m } (0.75 \text{ m}) \ (23.5 \text{ kN/m}^3)$

$w_A = 1.76$ kN/m / stringer

unit weight of treated wood is 6.0 kN/m^3

therefore $w_w = 6.0 \text{ kN/m}^3 \ (0.19 \text{ m } (0.65 \text{ m}) + 0.75 \text{ m } (0.14 \text{ m}))$

$w_w = 1.37$ kN/m / stringer

therefore maximum unfactored dead load moments are

$$M_{DL1} = \frac{w\ell^2}{8} = 1.76 \text{ kN/m} \frac{(7 \text{ m})^2}{8} = \underline{10.8} \text{ kN - m} \quad \text{Asphalt}$$

$$M_{DL2} = 1.37 \text{ kN/m} \frac{(7 \text{ m})^2}{8} = \underline{8.39} \text{ kN - m} \quad \text{Wood}$$

Dynamic Load Allowance "DLA"

Since the governing live loading is due to two axles of the design vehicle, but the span does not exceed 22.0 m, the DLA is to be taken as as 0.3 (2.4.3.2.7.)

FOR INFORMATION ONLY

For spans greater than 22.0 m, such as glued-laminated girders, the DLA is dependent upon the first flexural frequency (FFF) as outlined in Clause 2.4.3.2.4.

The FFF would be determined from the following equation:

$$FFF = \frac{\pi}{2L^2} \frac{EI}{m}$$

where L = span length (m)

E = modulus of elasticity girder = E_{50} x 106 (N/m^2)

I = moment of inertia per girder (m^4)

m = total mass carried by one girder kg/m

However for this bridge the DLA is 0.30 but must also be modified by 0.7, (2.4.3.2.10), for wood components.

therefore DLA = 0.3(0.7) = 0.21

Total Factored Load Effect "M_{TOT}"

The total factored load effect, per stringer, at ULS is;

$$M_{TOT} = \alpha_1 M_{DL1} + \alpha_2 M_{DL2} + \alpha_L \left(\frac{S}{D_d}\right) (M_{LL}) (1 + DLA)$$

where M_{DL1} = dead load asphalt = 10.8 kN - m

M_{DL2} = dead load wood = 8.39 kN - m

$\frac{S}{D_d}$ = load fraction = 0.48

M_{LL} = live load moment = 205 kN - m

DLA = dynamic load allowance = 0.21

therefore $M_{TOT} = 1.5(10.8) + 1.2(8.39) + 1.4(.48)\ (205)\ (1 + 0.21)$

M_{TOT} = 193 kN - m / stringer

Therefore: total factored load effect M_{TOT} = 193 kN - m per stringer and the factored resistance M_u = 231 kN - m per stringer therefore okay for flexure at ULS.

3.2.1.2. Shear

Factored Shear Resistance V_u

$$V_u = \frac{\phi f_{vu}\ bd}{1.5}\ k_d\ k_m\ k_v \qquad (13.7.1.1.)$$

where ϕ = 0.8 (Table 13.4.4.)

f_{vu} = 1.1 MPa (Table 13.11.2(b))

b = width stringer = 190 mm

d = depth stringer = 650 mm

k_d = 1.0 (13.5.3.)

k_m = 1.0 (13.5.6.)

k_v = size effect factor (13.7.1.2.)

For sawn wood k_v is a function of the length of possible end splits in the beam. The value specified for Select Structural is

$$k_v = \frac{45}{\sqrt{d}}$$

where d = depth of stringer

However when evaluating an existing bridge the end splits should be measured and the formula

$$k_v = \frac{75}{\sqrt{d}} \frac{1}{1 + 2\,(a/d)} \qquad (\underline{13.7.1.2.2.})$$

should be used. However, if the bridge is less than five years old k_v shall not be greater than that given for new structures. The values given for new bridges are based on the maximum permissible end splits allowed under the NLGA [3] grading rules.

As indicated in Clause 13.7.1.2.1., the select structural k_v factor may be used for No.1 grade material if the end splits safisfy the requirements for select structural. The advantage of this will be better understood if the two formulae are applied;

Select Structural $k_v = \frac{45}{\sqrt{d}} = \frac{45}{\sqrt{650}} = 1.77$

No. 1 grade $k_v = \frac{30}{\sqrt{d}} = \frac{30}{\sqrt{650}} = 1.18$

Since shear will normally govern sawn wood designs and the specified strength for shear is the same for both grades then the k_v factor is the only differentiation between the grades in shear. As applied in the resistance formula for V_u this would result in 50 percent larger sections when using No. 1 grade as opposed to select structural. (i.e. $\frac{1.77}{1.18} = 1.50$). Therefore, it is recommended the designer request that No. 1 grade material for beams and stringers meet the Select Structural requirements for end splits.

It should be noted that special grading requirements in addition to the NLGA rules are not uncommon. As well, seldom do end splits occur to the maximums allowed under the NLGA rules. In other words, the No. 1 grade material is usually graded as such, due to requirements other than end splits.

However, for this design $k_v = \frac{45}{\sqrt{d}} = 1.77$

therefore $V_u = 0.8 \frac{(1.1)}{1.5})190)\ (650)\ (1.77) = 128200$ N

Factored Shear Resistance V_u = 128 kN

Load Distribution (3.7.1.4.)

The application of the simplified methods of analysis for shear do not require the determination of the rigidities.

The distribution factor D_d is selected from Table 3.7.1.4.1. which provides a D_d = 1.70 for two lanes loaded.

However, for bridges with girder spacings less than 2.0 this must be modified using the following equation

$$D_d = \frac{(S)^{0.25}}{2.0} D$$

where S = spacing girders = 0.75 m

D = D_d value selected from Table 3.7.1.4.1.

D = 1.70 m

therefore final $D_d = \frac{(0.75)^{0.25}}{2.0} (1.70) = 1.33$ m

D_d = 1.33 m

Note: this distribution factor already includes modification for multi-lane loading (3.7.1.4.1.(c)).

The governing load fraction of one line of wheels of the OHBDC design vehicle (or half the design lane loading) is;

$$\frac{S}{D_d} = \frac{0.75}{1.33 \text{ m}} = 0.564$$

Live Load "V_{LL}"

The critical section for shear in wood members is to be taken at a distance from the support equal to the depth of the beam. (13.7).

Therefore the maximum shear at 0.65 m from the support is 243 kN* per design lane, therefore the unfactored live load shear per ½ design lane V_{LL} = 121 kN

Dead Load "V_{DL1}" and "V_{DL2}"

unfactored asphalt dead load w_A = 1.76 kN/m

unfactored wood dead load w_w = 1.37 kN/m

therefore unfactored dead load shears at 0.65 m from support are;

$V_{DL1} = w_A \left(\frac{\ell}{2} - 0.65\ m\right) = 1.76\ (3.5 - 0.65)$

V_{DL1} = 5.0 kN per stringer

$V_{DL2} = 1.37\ (3.5 - 0.65)$

V_{DL2} = 3.9 kN per stringer

Total Factored Load Effect "V_{TOT}"

The DLA and load factors are the same as for flexure.

Therefore $V_{TOT} = \alpha_1 V_{DL1} + \alpha_2 V_{DL_2} + \alpha_L \left(\frac{S}{DL}\right) (V_{LL})\ (1 + DLA)$.

therefore $V_{TOT} = 1.5(5.0) + 1.2(3.9) + 1.4(.564)\ (121)\ (1.21)$

V_{TOT} = 128 kN per stringer

Therefore; total factored load effect V_{TOT} = 128 kN per stringer
and the factored resistance V_u = 128 kN per stringer
therefore okay for shear at ULS.

* - calculations not included.

3.2.2. Serviceability Limit State I (SLSI)

Maximum allowable deflection "Δ_A"

$$\Delta_A = \frac{1}{400} \text{ of the span} \qquad (13.4.2.)$$

therefore $\Delta_A = \frac{7000}{400}$ mm = 17.5 mm

Live Load Deflection (SLSI) "Δ_L"

Section 3.4.5.3 allows the application of the simplified method of analysis to determine maximum deflections. The load fraction "$\frac{S}{D_d}$", which is determined from the simplified analysis, is the fraction of one line of wheels of the design vehicle which is applied to a width "S" of the bridge. The width "S" of the bridge is treated as a beam and the deflection is determined due to flexure only.

The SLSI, in contrast to the ULS, allows only one design vehicle on the bridge, which must be placed in the centre of the driving lane (2.4.3.1.7). Therefore a new $\frac{S}{D_d}$ must be determined for one lane loaded as outlined in Section 3.7.1.2.2.

Using the previous α and Θ values the D and C_f values are obtained from figure 3.7.1.2.2(b) for one lane loaded in two lane bridges.

$\alpha = 0.0489$ and $\Theta = 1.28$

therefore $D = 1.88$ and $C_f = 1.5$

$$D_d = D\left(1 + \frac{\mu C_f}{100}\right) = 1.88\left(1 + \frac{(-.05)(1.5)}{100}\right)$$

$$D_d = 1.88$$

The load fraction of one line of wheels of the design vehicle is

$$\frac{S}{D_d} = \frac{0.75}{1.88} = 0.40$$

NOTE: Section 3.7.1.2.2. also allows for further modification in cases where the outside line of wheels of the vehicle are more than 1.0 m from the free edge of the bridge. This is the case here, as for SLSI the vehicle must be placed in the centre of the driving lane. However, for a first check the value of $\frac{S}{D_d}$ derived above will be used.

Treating a width "S" of the bridge as a beam, which in this case is only one stringer, the maximum deflection due to one line of wheels of the SLSI loading is

$$\Delta = 16.0 \text{ mm}^*$$

Therefore the maximum deflection on the transverse section is

$$\frac{S}{D_d} \quad (\Delta) = 0.40 \ (16.0 \text{ mm})$$

$$\Delta_{max} = 6.5 \text{ mm} << \Delta_A = 17.5 \text{ mm}$$

Deflection usually only governs in longer span bridges such as Glued-Laminated girders.

*Calculation not shown. The two 140 kN axles govern.

4.0 EVALUATION OF EXISTING BRIDGES

The majority of existing wood bridges would have been designed to the WSD methods of a number of codes accepted in Ontario. In a recent calibration study [4] the OHBDC wood design specifications were compared against several of these WSD methods.

This calibration study indicated that the OHBDC for the most part produces designs requiring larger sections than the previous design methods. Considering that the OHBDC introduces impact (DLA) for wood bridges could in itself account for increased section sizes. However, as can be seen in Report 83-SRR-4, it is impossible to execute a direct comparison of the OHBDC's LSD method with that of previous WSD methods. In any case when applying the OHBDC specficiations to the evaluation of existing wood bridges the designer should proceed with caution.

4.1. Field Inspection

The OHBDC provides some guidelines for the evaluation of existing wood bridges (14.5.4.). A number of common ailments are discussed in the commentary (C14.5.4.) along with several methods of inspection. Field inspection should be considered as one of the most important parts of the evaluation process for wood bridges.

4.1.1. Decay

Caution is advised in the assessment of a structure for decay. Bridges constructed of pressure preservative treated wood using oil borne preservatives are believed to be the most resistant to decay attack. A number of bridges built as far back as the 1930's [5] in Ontario, are to-day still in sound condition and display few if any signs of decay. Members which are pressure treated, as opposed to simple dip treatment* are usually distinguishable by small incisor marks on the surface. These cuts allowed the treatment to penetrate deep into the wood. However the lack of marks does not mean the members were not pressure treated.

*Dip treatment using a bath has not been allowed in Ontario for many years.

Softening of a member does not necessarily mean decay exists as the oil treatments themselves tend to soften the wood surface. In addition, sounding of a member for decay pockets can also be misleading, as members under load (i.e. dead load) will often sound hollow. In most cases where a structure is particularly susceptible to decay, for example due to improper treatment or severe splitting or checking, it usually will display visible signs of decay. Though mild probing may be beneficial in local areas such as where water becomes trapped or where members are in ground contact, the driving of nails or other objects is not recommended. If the bridge displays visible signs of decay or it is suspected that hidden decay exists it is suggested that an experienced wood investigator be called to determine the extent of decay.

4.1.2. Material Identification and Grading

According to the OHBDC, unidentified wood shall be taken as Spruce-Pine-Fir (SPF) and since no grade is specified one must assume No.1. However the majority of wood bridges supporting larger size structural timbers are usually composed of West Coast Douglas Fir (DF), which is in most cases Select Structural grade. The difference in specified strengths for S-P-F No. 1 and Select Structural Douglas Fir are as follows:

S-P-F	D.F.
f_{bu} = 13.0 MPa	f_{bu} = 24.0 MPa
F_{vu} = 0.8 MPa	f_{vu} = 1.1 MPa
E_{50} = 11000 MPa	E_{50} = 9 000 MPa

It is evident that there is a great advantage in identifying and grading the material in an existing bridge. In the case of the Sioux Narrows bridge [5] the material was identified as Dense Select Structural Douglas Fir. Though the OHBDC does not identify with dense material, the designer can reference previous specifications which will enable him/her to apply some rational engineering judgement as to the increased strength and stiffness that may be considered.

Species identification can also be performed by investigators unfamiliar with wood bridges, by obtaining several samples of the bridge material for laboratory analysis. Several organizations in Ontario provide this analysis service. Even without grading, if the species is identified as Douglas Fir the assumed specified strengths are increased considerably. The shear strength is the same for select and No. 1 grades, except for the effects of end splits, which was discussed previously in Section 3.2.1.2. of this report.

4.1.3. End Splits and Checks

It is necessary to measure end splits in existing beams and stringers in order to apply the ULS shear requirements of the OHBDC. It will probably be too conservative to assume the conditions provided for new structures, as seldom do splits occur which are as long as those allowed under the NLGA grading rules. The methods for applying these end splits is outlined in Section 13.7.1.2.2. of the Code.

The identification and measurement of end splits is important as they can easily be confused with checks and shakes. Though consultation of the NGLA grading rules is recommended, the basic difference is that splits penetrate through the member from one face to the other, where as checks and shakes only penetrate from one side. Though normally checks and shakes are not considered to affect the strength, the engineer should apply rational judgement dependent upon their size and extent.

The length of end splits, with respect to the OHBDC requirements, should be measured from the centreline of the supports. The maximum effect on shear strength is when the split occurs at midheight and therefore judgement is necessary when splits occur near the top or bottom of a flexural member. The engineer may wish to apply the NLGA requirements, which indicate that splits, checks and shakes, outside the middle $\frac{1}{2}$ of the height of the beam are not considered.

4.2 Conclusions on Evaluation

The design specifications of the OHBDC should be applied to the evaluation of an existing wood bridge, in the same way as for new structures except for the effects of end splits.

A thorough field inspection should be performed to determine the extent of decay, as well as measure end splits and section sizes. Where possible the material should be identified and graded.

It is recommended that engineers who frequently become involved in wood bridge evaluation obtain a copy of the NLGA grading rules. These rules identify the many accepted abnormalities associated with structural wood, which could concern the inexperienced investigator. They also provide detailed description of the grades, which allow some judgement of the material quality for an initial evaluation. However proper identification and grading should be performed by experienced personnel.

5.0 CONCLUSIONS

This report is meant to familiarize the designer with the new OHBDC wood design specifications. It is recommended that the designer also become familiar with the commentary which provides extensive background on the developments of the specifications.

The interpretation of the specifications, other than those of Section 13, expressed in this report should not be considered binding.

It is expected that in the near future a great number of existing wood bridges will require evaluation. The importance of the field investigation should be foremost. There are a number of excellent references for the evaluation of wood structures, the latest of which was published by the ASCE [6] in 1982 on the "Evaluation, Maintenance and Upgrading of Wood Structures".

6.0 REFERENCES

[1] "The Ontario Highway Bridge Design Code", MTC., Toronto, Ontario, 1979.

[2] "The Ontario Highway Bridge Design Code", MTC., Toronto, Ontario, April 1984.

[3] "Standard Grading Rules for Canadian Lumber", National Lumber Grades Authority, Ganges, B.C. Canada, September, 1979.

[4] "Wood Bridge Calibration Study for the OHBDC", Taylor, R. J., MTC., Toronto, Ontario, 1983. MTC Report 83-SRR-4.

[5] "Prestressed Wood Bridges", Taylor, R. J., Batchelor, B. deV., Van Dalen, K. Reprint CSCE International Conference on Short and Medium Span Bridges, Toronto, Ontario, 1982. MTC Report 83-SRR-1

[6] "Evaluation, Maintenance and Upgrading of Wood Structures, A. Guide and Commentary", ASCE, New York, N.Y. 1982.